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Geo-engineering Investigation and Evaluation of a Proposed Construction Site in Apapa Area, Southwestern Nigeria

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Abstract

Geotechnical investigation of the subsurface was carried out in a proposed six storey building site at Barthrust Road, Apapa, Lagos aimed at determining the stratigraphy and competency of the shallow formations as foundation material. Field investigation techniques such as borings, Standard Penetration Test and Cone Penetration Test were carried out to fulfill the objective. A total of 3 boreholes were drilled upto depth of 30.0m while Cone penetration Test was carried out upto depth of 3.25m. Samples from boreholes were sent to laboratory for sieve analysis, Atterberrg limits, un-drained triaxial compression and consolidation tests. Boring and penetration tests revealed that the shallow subsurface is underlain essentially by loose silty sand (0.0 – 1.50m) of friction angle 28^0 and allowable bearing capacity of $29.0kN/m^2$ overlying medium dense silty sand (1.50 – 14.50m) with angle of internal friction of 31^0. Beneath these layers is soft silty sandy clay (14.50 – 16.0m) with cohesion of $35.0kN/m^2$ and angle of internal friction of 5^0 overlying a medium dense silty sandy layer (16.0 – 21.0m) with angle of internal friction of 32^0. These are further underlain by dense silty sandy layer (21.0 – 30.0) with an angle of internal friction of 34^0 which is the most competent foundation material. Grain size distribution analysis revealed that cohesion less soil have d_{10}, d_{30}, d_{60}, C_u, and C_c range of 0.07 – 0.20, 0.18 – 0.80, 0.22 – 1.50, 1.6 – 21.4 and 0.50 – 2.40 respectively. Samples taken at depths approximately 0.0 – 19.0m are classified as poorly graded pinpointing their susceptibility to liquefaction while those at depths 19.0 – 30.0m are well graded. The allowable bearing capacity of the shallow foundation zone from the topsoil to 1.50m shows low bearing capacities characteristics which are grossly lower than the projected foundation loading. Pile driven to competent layer at 18.0m is recommended as foundation option for consideration for the proposed structure.

Keywords: Boring, Geotechnical, Standard Penetration Test, Foundation, Triaxial test, Bearing capacity, Apapa, Southwestern Nigeria.

Introduction

Site investigation and estimation of soil characteristics are essential parts of a geotechnical design process. It aid in detailed understanding of the engineering and geologic properties of the soil, rock strata and groundwater conditions that could impact the safe design of structures. It is a prerequisite for the successful and economic design of engineering structures and earthworks (Anon, 1999). Exploration of the subsurface can be done using remote geophysical methods and borings/penetration tests. The boring though expensive is more reliable and helps in recovering samples for laboratory testing and evaluation (Coduto, 2001). However, in-situ testing is very important in geotechnical engineering, as simple laboratory tests may not be reliable while more sophisticated laboratory testing can be time consuming (Mair and Wood, 1987). Among in-situ testing methods is the Standard and Cone Penetration Tests. Standard Penetration Test is used to identify soil type and stratigraphy along with being a relative measure of strength while Cone Penetration Test (CPT) allows for the soil type to be determined from the measured values of cone resistance and sleeve friction and provide a record of soil properties with depths. Sanglerat (1972) and De Ruiter (1981) reviewed the application of Cone penetration testing in geotechnical practice and concluded that it can be utilized for a wide range of geotechnical engineering applications. Detailed description and interpretation of the SPT is given elsewhere (e.g., Seed et al., 1975; Liao and Whitman, 1986; Clayton, 1995). SPT, with its ease of performance and extensive correlation with parameters used in foundation design is the prevalent method in evaluating the allowable bearing stress for foundation of engineering structures. SPT is used to determine the density of granular strata and correlate the undrained shear strength of cohesive soils. It is a frequently used and accepted method of empirically determining soil strength and calculating the bearing capacity and settlement of granular soils. SPT data have been used in correlations for unit weight, relative density, angle of internal friction and unconfined compressive strength (Kulhawy and Mayne, 1990). Zekkos et al., (2004) studied the reliability of shallow foundation design using SPT test. The results of reliability analysis show that the factor of safety approach can provide an impression of degree of conservatism that is often unrealistic and therefore submitted that the reliability based approach using SPT-N provides rational design criteria, accounting for all key sources of uncertainty in the foundation engineering process and should be the basis of design. Lutenegger (2008) showed that the SPT provides three numbers that can be used to evaluate soil properties through an analysis to illustrate how the incremental blow counts may be used to obtain more information from the test. The Standard Penetration Test (SPT) technique can provide much of the information required during a site investigation as compared to other field techniques (Wazoh and Mallo, 2014). This study will therefore place preference on SPT method in estimating the allowable bearing capacity. Despite the increasing use of in-situ methods in geotechnical design process, there is still need to recover samples for laboratory analysis which will help in simulation of field worst conditions. Therefore, integration of field and laboratory methods as used in this study may greatly improve the stability and quality of engineering construction as it will capture all necessary data required for safe design of structures. The focus area of this work is Barthrust Road, G.R.A

in Apapa, Lagos, Nigeria (Fig. 1) which is a site of a proposed six storey building. It is a lowland area located within latitudes $6^{0}27^{'}$- $6^{0}450^{'}$N and longitudes $3^{0}22^{'}$ – $3^{0}367^{'}$E with an elevation of 2 to 3m above the sea level. This study determines the stratigraphy and competency of the Quaternary deposits underlying the area as well as the relevant engineering characteristics of the deposits to enable appropriate foundation design for the proposed structure.

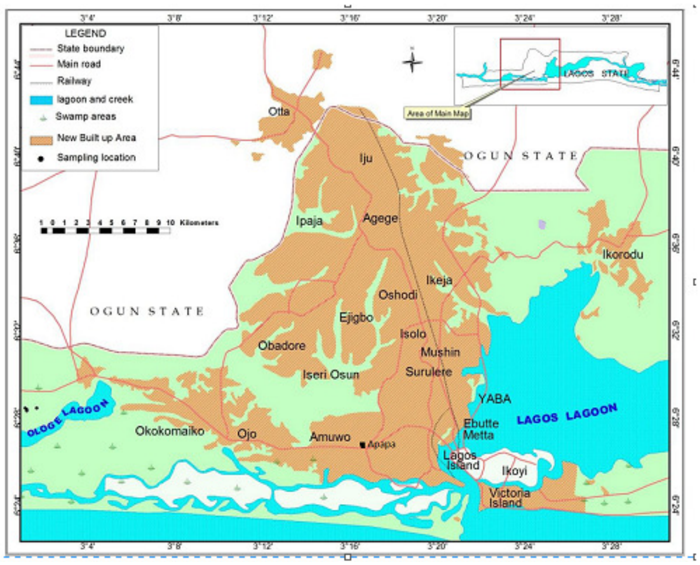

Fig. 1. Map of Lagos showing location of the study area (Modified after Ogundele, 2012)

Geology of the Study Area

Lagos metropolis is the area of land around the only inlet of the sea into the extensive lagoon system. It falls extensively within Dahomey Basin. The Basin is a combination of inland/coastal/offshore sedimentary basin in the Gulf of Guinea (Obaje, 2009). The lithology based stratigraphic classification of Dahomey basin by Jones and Hockey (Brownfield and Charpentier, 2006) is suitable for this study since lithology is a key parameter in determining suitability of materials for engineering purposes. Stratigraphically, the basin is divided into Abeokuta Formation, Ilaro Formation, Coastal Plain Sands and Recent Alluvium sediments (Jones and Hockey, 1964). Deposition of Cretaceous sequence in the eastern Dahomey Basin began with the Abeokuta Group, consisting of the Ise, Afowo and Araromi Formations (Omatsola and Adegoke 1981). The Ise Formation, the oldest, unconformably overlies the basement complex and consists of conglomerates and sandstones at base and in turn overlain by coarse to medium grained sands with interbedded kaolinite. Overlying the Ise Formation is the Afowo Formation, which is composed of coarse to medium grained sandstones with variable but thick interbedded shales, siltstones and claystone. The Araromi Formation overlies the Afowo Formation and is the youngest Cretaceous sediment in the eastern Dahomey basin (Omatsola and Adegoke, 1981). It is composed of fine to medium grained sandstone overlain by shales, siltstone with interbedded limestone, marl and lignite. The Ewekoro Formation, an extensive limestone body, overlies the Araromi Formation. The Ewekoro Formation is overlain by the Akinbo Formation, which is made up of shale and clayey sequence. Overlying the Akinbo Formation is Oshosun Formation which consists of greenish – grey or beige clay and shale with interbeds of sandstones. The Ilaro Formation overlies conformably the Oshosun Formation and consists of massive, yellowish, poorly, consolidated, cross-bedded sandstones. The Quaternary sequence in the eastern Dahomey basin is the Coastal Plain Sands and recent littoral Alluvium (Durotoye, 1975) and consists of poorly sorted sands, silts and clay deposits with traces of peat in parts. The sands are in parts crossbedded and show transitional to continental characteristics. The age is from Oligocene to Recent. It directly underlies the study area and is composed of deposits which can be divided into the littoral and lagoonal sediments of the coastal belt and the alluvial sediments of the major rivers. They consist predominantly of unconsolidated

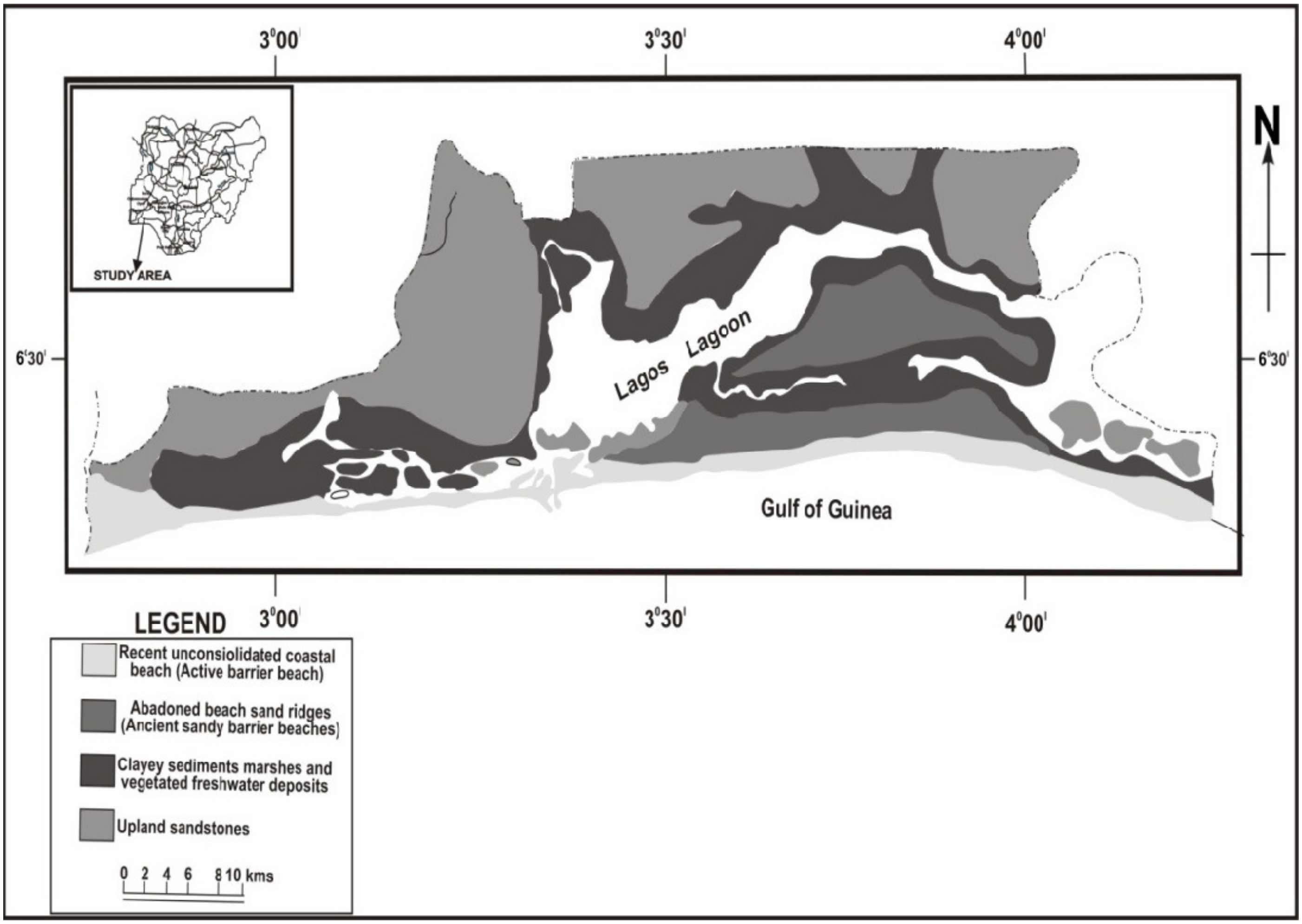

Fig. 2 Map of surface geology and morphology of Lagos (Adebisi and Fatoba, 2013).

sands, clays and mud with a varying proportion of vegetative matter (Fig. 2).

Materials and Methods

1. Boring and Standard Penetration Testing (SPT)

Three boreholes were drilled to depth 30.0m each with 250mm, 200mm and 150mm diameter steel casings using a Percussion motorized Shell and Auger rig employing light cable percussion boring techniques with a fully equipped motorized Pilcon Wayfarer drilling rig. The position of borehole 1, 2 and 3 coincide with that of CPT P_2, P_4 and P_6 respectively. CPT P_1 to P_2 is approximately 20.0m apart on a straight line such that the distance between CPT P_1 to P_7 is 120.0m. During drilling operations, disturbed soil samples were regularly taken at depth interval of 0.75m and whenever a change of stratum is observed. In cohesive soil strata, apart from the usual disturbed samples, undisturbed samples were taken using the conventional open tube sampler by driving a 100mm diameter sampler through a total of 450mm length. All samples comprise those from the split spoon of standard penetration test and those of the cutting shoe of 100mm diameter sampler. All disturbed (D) and undisturbed (U) samples recovered from the borehole were examined, identified and classified in the field. They were later taken to the laboratory for detailed investigations where a total of 22, 6, 3 and 2 samples were subjected to grain size distribution test by wet sieving, Atterberg limit test,

quick undrained triaxial test and consolidation by Oedometer test respectively. Effort was made to ensure that all strata encountered were tested appropriately. Also, Standard Penetration Test (SPT) was carried out at 1.5m intervals in both cohesive and cohesion less soils with disturbed samples recovered from the SPT sampling tool. In carrying out the SPT test, a 50mm diameter split spoon sampler is driven into the soil using a 63.5kg hammer with a 760mm drop, and the penetration resistance is expressed as the number of blows (SPT-N-value) required obtaining a 300mm penetration below an initial 150mm penetration seating drive. The SPT-N-values were corrected for borehole and dilatancy where necessary and all pertinent borehole data, penetration resistance, and sample data were recorded on the boring log sheet (Fig. 7 – 9). In boreholes, SPT results are routinely used to provide an estimate of density, consistency, unconfined compressive strength and shear strength parameters. The SPT-N value is assumed to be dependent on relative density in granular soils and undrained shear strength in cohesive soils. The SPT results were used to determine the cohesion and angle of internal friction of some strata in tandem with Sanglerat, (1972) approach.

2. Cone Penetration Testing (CPT)

Cone penetration testing can be utilized for a wide range of geotechnical engineering applications. Sanglerat (1972) and De Ruiter (1981) reviewed the application of the method in geotechnical practice. The Cone Penetration Test is a means of ascertaining the resistance of the soil. Seven CPT tests were carried out to a depth approximately 3.0m. The tests were performed using a 2.5-Ton nominal capacity manually powered CPT machine. Penetration resistance (qc), sleeve friction (fs) and the depth of penetration were recorded at each station (Table 1). Most of the test reached refusal before the anchors pulled out of the subsurface. The layer sequences were interpreted from the variation of the values of the cone resistance with depth. On the basis of the expected resistance contrast between the various layers, inflection points of the penetrometer curves were interpreted as the interface between the different lithologies or density variation. The cone penetration test is economical and supplies continuous records with depth.

Table 1. Cone penetration test data

	Cone Readings(kg/cm^2)						
Depth(m)	P_1	P_2	P_3	P_4	P_5	P_6	P_7
0.25	5	3	2	3	3	3	3
0.50	2	2	2	6	5	2	2
0.75	7	2	2	5	5	2	5
1.00	10	2	5	9	7	6	8
1.25	22	8	10	12	10	10	12
1.50	36	30	40	26	15	26	20
1.75	66	50	44	40	28	34	36
2.00	80	55	59	50	42	50	46
2.25	92	64	68	62	58	68	64
2.50	104	72	80	74	70	80	70
2.75	110	77	84	88	84	84	88
3.00	-	85	106	110	94	88	106
3.25	-	95	-	-	116	106	-
3.50	-	-	-	-	-	-	-
3.75	-	-	-	-	-	-	-
4.00	-	-	-	-	-	-	-

3. Particle Size Distribution by Wet Sieving Testing

Sieve analysis was conducted on 22 samples in the laboratory in accordance with BS 1377 (1990) using 65gm of air dried samples to determine the particle size distribution. Aggregation of the particles was broken by mortar and rubber covered pestle. The grain size distribution was conducted using a set of US standard sieves (No. 4, 10, 20, 40, 60, 100, 200 and pan). A lid was also placed at the top to provide cover of the sample. Weight of each sieve was determined before staking. Stack of sieves were shaken by mechanical sieve shaker. The stacks of

sieves were removed at 5 minutes interval. Combined weight of each sieve and sample was measured. Wet washing was conducted to prevent aggregation of large clumps of fine particles in soil samples retained on sieve No. 200. A bowl was placed under the sieve. Washing of sample was continued until clean water was coming out. Remaining sample was dried in the oven and weight was measured.

Table 2. Grain size distribution results parameters

Serial No	Sample Designation	Depth (m)	Effective size (d_{10})	Effective size (d_{30})	Effective size (d_{60})	Coefficient of Uniformity (C_u)	Coefficient of Curvature (C_c)
1	Apa1 B1	0.75	0.10	0.18	0.25	2.5	1.3
2	Apa2 B1	4.50	0.08	0.20	0.40	5.0	1.3
3	Apa3 B1	12.00	0.15	0.18	0.22	1.6	1.0
4	Apa6 B1	18.00	0.15	0.20	0.50	3.3	0.5
5	Apa1 B2	1.50	0.15	0.40	0.50	3.3	2.1
6	Apa2 B2	6.00	0.20	0.30	0.55	2.8	1.8
7	Apa4 B2	15.00	0.20	0.40	0.70	3.5	1.1
8	Apa5 B2	20.25	0.20	0.60	1.50	7.5	1.2
9	Apa6 B2	29.25	0.20	0.60	1.50	7.5	1.2
10	Apa1 B3	0.75	0.20	0.40	0.50	2.5	1.6
11	Apa2 B3	7.50	0.15	0.20	0.35	2.3	0.8
12	Apa4 B3	19.50	0.20	0.80	1.50	7.5	2.1
13	Apa5 B3	24.00	0.07	0.50	1.50	21.4	2.4

4. Atterberg Limit Testing

The tests were carried out in accordance with BS 1377(1990). Soil Samples passing through No. 40 sieve were used in the test. Casagrande Liquid limit device and the grooving tool was cleaned as well as fall height (1 cm) of the cup was adjusted. Appropriately, 250 gm soil samples were taken in a bowl and mixed with water. Water content of 25% was considered in the first trial. After addition of water, the soil sample was chopped, stirred and kneaded repeatedly. A portion of the soil was placed in the device. A groove was cut at the center of the placed soil in the device. The cup of the device was lifted and dropped by a rate of 2 drops/second. The process was continued until the groove was closed around 13 mm. The test was repeated for three times to plot no of blow against moisture content. Liquid limit was the moisture content corresponding to 25 blows on the straight line. For Plastic limit, soil samples were separated in the plate. Ellipsoidal soil masses were formed by adding water. Soil masses were rolled in the glass plate until they became threads of about 3 mm. When the threads were broken at 3 mm diameter, they were taken in the moisture cans. Samples were dried in the oven and moisture contents were determined.

Table 3. Atterberg Limit Test Results

S/N	Sample No	Depth (m)	Natural Water Content(%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index(%)	Liquidity Index (%)	Swelling Potential (%)
1	Apa8 B1	15.00	28	46	18	28	0.36	7
2	Apa9 B1	14.25	25	36	21	15	0.27	2
3	Apa7 B2	15.00	24	44	17	27	0.23	7
4	Apa8 B2	15.50	26	43	20	23	0.26	5
5	Apa6 B3	15.00	28	40	21	19	0.37	3
6	Apa7 B3	15.50	24	38	19	19	0.26	3

5. Oedometer Consolidation Testing

The specimens were loaded and unloaded in several steps. At each loading stage the change of the height was recorded at suitable intervals while consolidation takes place. At the end of the test, the final dial gauge readings were taken. After removing the dial gauge and the top plate, the measurements of the final height of the specimen were determined by the calipers. Immediately after that, the free water was removed from the soil surface and specimen weighed. The water content and void ratio were then

Table 4. Oedometer Test Results

Sample/ No	Depth (m)	Bulk Density (Mg/m^3)	Pressure Range (KN/m^2)	Initial Void Ratio	Coefficient of Volume Compressibility M_v (m^2/MN)	Coefficiention of Consolidation (C_v) (m^2/year)
Apa10B 1	15.00	1.85	0-25	0.810	0.714	2.3
			25-50		0.181	2.0
			50-100		0.097	1.5
			100-200		0.069	1.3
			200-400		0.050	1.0
Apa9B2	15.50	1.71	0-25	0.840	0.575	5.2
			25-50		0.163	1.7
			50-100		0.100	1.2
			100-200		0.071	1.6
			200-400		0.045	1.2

determined. The compressibility and cohesive values were compared with the guidelines proposed by Bell (2007).

6. Undrained Triaxial Compression Testing

Axial loading is commenced immediately after the chamber pressure $ð_3$ is stabilized; a cylindrical specimen of soil encased in a rubber membrane was placed in a triaxial compression chamber, subjected to a confining fluid pressure and then loaded axially to failure. Connections at the ends of the specimen permit controlled drainage of pore water from the specimen. Connections at the ends of the specimen permit controlled drainage of pore water from the specimen. The three principal stresses were known and are controlled. The three principal stresses were known and are controlled. Prior to shear, the three principal stresses were equal to the chamber fluid pressure. During shear, the major principal stress, $ð_1$ equalled the applied axial stress (P/A) plus the chamber pressure, $ð_3$. The applied axial stress or deviatorial stress ($ð_2$), was calculated as the difference between $ð_1$ and $ð_3$. The deviatorial stress was then plotted against the principal major stress ($ð_1$) to generate Mohr envelope where the angle of internal friction and cohesion was derived. Prior to shear, the three principal stresses were equal to the chamber fluid pressure. During shear, the major principal stress, $ð_1$ equalled the applied axial stress (P/A) plus the chamber pressure, $ð_3$. The applied axial stress or deviatorial stress ($ð_2$), was calculated as the difference between $ð_1$ and $ð_3$. The deviatorial stress was then plotted against the principal major

Table 5. Undrained Triaxial Test

S/No	Depth (m)	Sample Designation	Natural Water Content (%)	Bulk Density (Mg/m^3)	Principal Minor Stress (KN/m^2)	Principal Major Stress (KN/m^2)	Cohesion (KN/m^2)	Angle of Internal Friction(^{0})
6	15.00	Apa11B1	28	1.85	150	260	34	6
					200	320		
					250	380		
7	15.50	Apa10B2	26	1.76	150	280	35	5
					200	340		
					250	400		
8	15.50	Apa8B3	24	1.58	150	250	38	4
					200	305		
					250	370		

stress ($ð_1$) to generate Mohr envelope where the angle of internal friction and cohesion was derived. In this multi-stage element, a single specimen was compressed at three effective stress stages, rather than using the more familiar three individual specimens. The reason for using the multi-stage approach is that fewer samples require less time in the field, and that issues of non-uniformity between samples were removed.

The poisson ratio and modulus of elasticity were evaluated from the method of Bowles (1997). The angle of internal friction was estimated from penetration tests based on Sanglerat (1972), Bowles (1997) and Meyerhof (1965) methods.

Plasticity Index (PI) = LL – PL, where LL = liquid limit, PL = plastic limit

$$\text{Liquidity Index (LI)} = \frac{w - PL}{PI}$$

Where w = Natural water content, PL = plastic limit, PI = plasticity index.

The Swelling Potential (S_p) was estimated using Seed et al (1957) approach; $S_p = 60k \times I_p^{2.44}$ where $k = 3.6 \times 10^{-5}$.

Teng (1969) equation was used to estimate the net allowable bearing capacity under shallow foundation for 40.0mm permissible settlement.

$$\text{Net allowable capacity } (q_{na}) = 55(N-3)\frac{(B+0.3)^2}{(2B)^2} \times k_d \times R_w \text{ ....(i)}$$

Where q_{na} is the net allowable bearing capacity, N is the average SPT-N value of the bearing stratum at base of the foundation, B is the width of the foundation, k_d is the depth correction factor, R_w is the water table correction factor.

Using the equation of ultimate bearing capacity for a driven pile;

$Q_f = q_f \times A_b + f_s \times A_s$ where

Q_f = ultimate load that can be applied at the top of the pile,

q_f = ultimate bearing capacity of the stratum on which the pile is supported,

f_s = the average shearing resistance of soil per unit area,

A_b = area of the pile at the base,

A_s = cylindrical surface area of the pile.

SPT-N method was employed using Meyerhof (1976) equation for driven pile;

$$q_f = 40 \times N \times \frac{D}{B} \text{ (limited by 400N) ....(ii)}$$

$f_s = 2N_a$ where

N is the SPT-N at the vicinity of the base of the pile,

N_a is the average SPT-N value over the embedded depth of the pile.

Since the above equation is applicable for driven piles, the value of q_f obtained was further multiplied by 0.33 while that of f_s was multiplied by 0.5 to derived corresponding values for bored piles.

Allowable bearing capacity:

$$q_a = q_u/F.S \text{ ....(iii)}$$

where F.S. is factor of safety = 2.5.

Results and Discussion

The results of various tests and their interpretation are explained below.

1. Soil Stratigraphy

The soil type interpreted from the indication of the penetration test and revealed during the drilling of the boreholes are generally consistent with slight variation in thickness and engineering properties across the site. Based on the results of physical logging of sub soil types and interpretation of laboratory tests, a typical soil profile characterizing the site area is described below and also shown in the borehole logs (Fig. 7 – 9).

(i) A very loose dark grey Silty Sand (0.0 – 1.50m)
(ii) A medium dense grey Silty Sand (1.50 – 14.50)
(iii) Soft light grey Silty Sandy Clay (14.50 – 16.0)
(iv) Medium dense Silty Sand (16.0 – 21.0)
(v) Dense light grey Silty Sand (21.0 – 30.0m)

The boreholes revealed that the shallow subsurface from the topsoil to 1.5.0m is very loose to loose Silty Sand. Besides, there is sandwiching of a bedded soft light grey Silty Sandy Clay layer between approximately 14.0 – 16.0m beneath the existing ground level. Groundwater seepage was encountered at an average depth of 0.75m below the existing ground level. The equilibrium water level could not be established in the borehole since some part of the investigated site was waterlogged at the time of carrying out fieldwork.

2. Penetration Tests

The depth probed by the penetrometer varied between 2.75 – 3.75m. The petrometer signatures showed that the resistance values ranges from 2-116kg/cm^2 (Table 1). The shallow subsubface soil from the topsoil to approximately 1.0m exhibits resistance ranging from 2 – 10kg/cm^2 interpreted to be very low since it is <20kg/cm^2 (Meyerhof, 1965). From 1.0 – 1.50m, the Silty Sand have resistance ranging from 10 – 40kg/cm^2 interpreted to be low since it is <50kg/cm^2, it can therefore be suggested that the shallow subsurfac soil from the topsoil to approximately 1.5m are poor foundation material. The low resistance values are often indications of soft layers or loose granular materials (Sanglerat, 1972). However, borehole data revealed that the subsoils are essentially made up of very loose dark grey Silty Sand from topsoil to 1.5m, this is consistent with Onwuka (1990) who attested to the great variability in the sedimentary deposits within Lagos Metropolis. It is important to mention that the relatively low SPT-N values of the Silty Sand is a pointer to it susceptibility to liquefaction especially under sudden load and submerged condition. In term of suitability as foundation material, the top 1.5m are probably unsuitable as foundation soil. Also, the SPT-N data revealed that the top 1.5m in the area have an average SPT-N-value of 4 (Fig. 7 – 9) which could be interpreted to mean that the soil up to that depth is very loose (Bell, 2007) and which could be inimical to the foundation of engineering structures in the area. The corroboration of both Standard Penetration and Cone Penetration Tests further confirm the unsuitability of the top 1.5m as foundation materials and also pinpoint the near accuracy of both methods in characterizing subsurface soil for engineering applications. The medium dense light grey Silty Sand (1.50 – 14.50m) with SPT-N values ranging from 13 – 21 is indicative of relatively moderate to medium density which qualifies it as moderately good foundation materials. However, depths 14.50 – 16.0m is underlain by soft light grey Silty Sandy Clay with SPT-N-value of 4 interpreted to be soft (Meyerhof, 1965), this layer could pose serious

danger to engineering construction especially when punctured by deep foundation in form of piles. Hence, pile foundation should not be founded on or near the layer since even a thin sensitive clay layer can lead to amplification in compressibility and reduction in the strength of an otherwise good material (Oyedele et al., 2011). Beneath the soft clay layer is a medium grained silty sand layer with an average N-values of 21 interpreted to be medium dense (Bell, 2007) which is competent enough to withstand appreciable foundation loads in form of piles. The dense light Grey Sand (21.0 – 30.0m) at the base with an average SPT-N value of 30 (Fig. 7 – 9) is the most competent strata investigated and will mobilize the highest foundation pressure when foundation is founded on it.

3. Classification Tests

3.1 Grain Size Distribution

The result of grain size distribution of 22 samples analyzed (Table 2 and Fig. 3 – 5) revealed that the effective size (d_{10}) ranges from 0.07 – 0.20mm, d_{30} ranges from 0.18 – 0.8mm while d_{60} ranges from 0.22 – 1.50mm. Also, the coefficient of uniformity (C_u) ranges from 1.60 – 21.4 while coefficient of curvature (C_c) range from 0.50 – 2.40. It is observed that out of the 13 samples whose effective diameters were estimated, four samples picked at depths approximately 19.0 – 24.0m (Apa5B2, Apa6B2, Apa4B3 and Apa5B3) have their coefficient of uniformity and curvature within set limit for well graded soil while samples retrieved at shallower depths <19.0m have their coefficient of uniformity and curvature fall within poorly graded soils. Hence, the soils taken at depths < 19.0m are essentially poorly graded since they all have their coefficient of uniformity less than 6 while those picked at depths >19.0m are well graded since their coefficient of uniformity is greater than 6 while their coefficient of curvature fall within 1.0 – 3.0 (Wagner, 1957) and (Casagrande, 1959). This is consistent with Onwuka (1990) who reported that Lagos metropolis is underlain by sedimentary deposits which exhibit high degree of variability and is essentially poorly graded. Grain size distribution curves of the samples (Fig. 3 – 5) also showed that the samples are predominantly gap graded pinpointing their relative susceptibility to liquefaction and poor compaction properties which is undesirable in engineering applications.

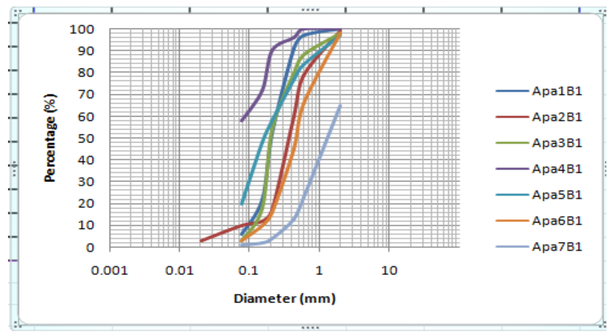

Fig. 3. Grain size distribution curves of samples from borehole 1.

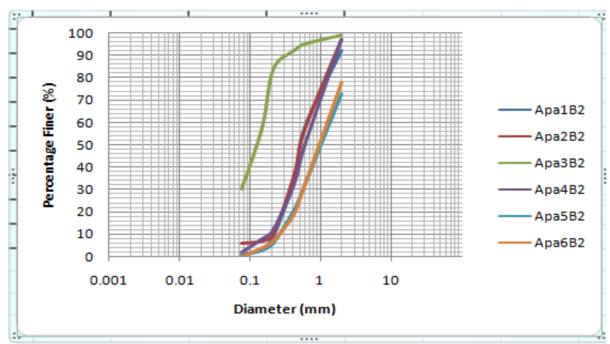

Fig. 4. Grain size distribution curves of samples from borehole 2.

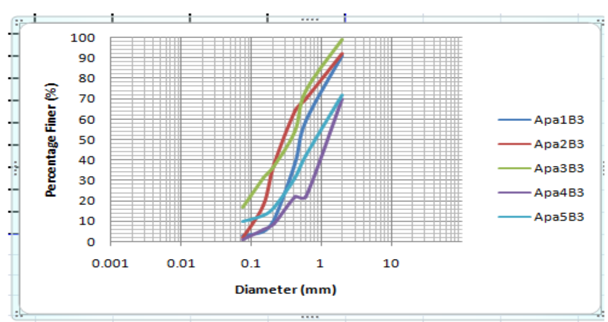

Fig. 5. Grain size distribution curves of samples from borehole 3.

3.2 Atterberg Limit Tests

The Atterberg consistency test results of 6 clayed samples taken at depths approximately 14.0 – 16.0m shows that the natural water content, liquid limit, plastic limit, plasticity index, liquidity index and swelling ratio ranges from 24.0 – 28.0%, 36 – 46%, 17 – 21%, 15 – 28%, 0.26 – 0.37 and 2 – 7% respectively (table 3). Plasticity chat (Fig. 4) shows that the soil falls essentially on clay and silts of medium plasticity indicating their medium potential compressibility behaviour. Also, the low liquidity index of the clay (<1%) is indicative of their relative insensitivity. Furthermore, the swelling potential index of the clay which is expected to be medium however shows medium to high potential swelling behavior (>5%) which is indicative that a medium plasticity clay may have surprisingly high swelling potential (Seed et al., 1957).

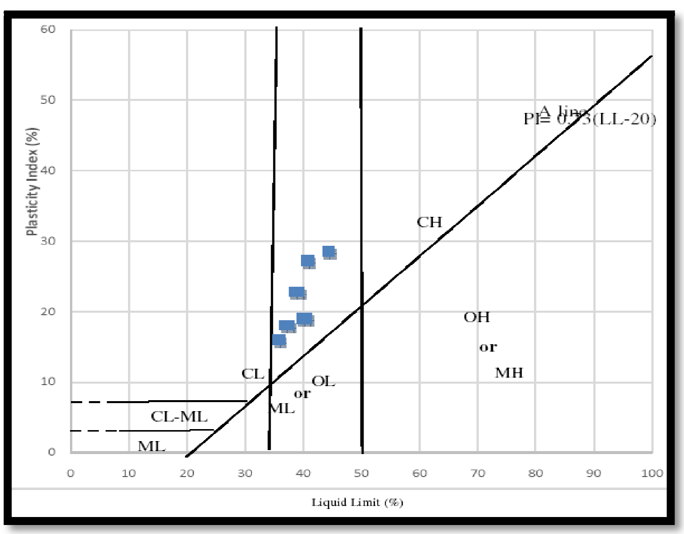

Fig. 6. Plasticity chart (Modified after Casagrande, 1932).

3.3 Oedometer Test

Based on the analysis of variation in equilibrium void ratio for various values of effective stress and the analysis of variation of dial gauge readings at various time intervals for a particular stress level with respect to square root of time, the coefficient of volume compressibility (M_v) and coefficient of consolidation (C_v) values were determined over a range of pressures for the clay stratum at depths approximately 14.0 – 16.0m. The coefficient of volume compressibility and consolidation of 0.045 – 0.714m^2/MN and 1.0 – 5.2m^2/year respectively are typical of low to high compressibility which is not competent enough to withstand loadings from a six storey building. Also, the sharp decrease in compressibility from 0.714 to 0.181m^2/MN for sample Apa10B1 and 0.575 – 0.163m^2/KN for sample Apa9B2 under incremental pressures of 25 – 50KN/m^2 and high initial void ratios of 0.810 – 0.840 is suggestive of higher initial settlement under load followed by a more gradual consolidation settlement.

3.4 Undrained Triaxial Compression Tests

Based on the variation between deviatorial stress and the height of the soil samples using the principle of effective stress, the shear strength parameters of samples taken from depths approximately 14.0 – 16.0m were determined. A range of cohesion values of 34kN/m^2 – 38kN/m^2 (Table 5) obtained for the samples are indicative of soft to firm clays. The value of angles of internal friction ($4^0 – 6^0$) is accounted for by the presence of sands and silts in the essentially clayey deposit. These strength parameters are indicative of low strength foundation founding material thereby rendering the stratum unsuitable for the proposed structure.

Table 6. Summary of Engineering Properties

Stratum Depth Range(m)	Thickness(m	Geotechnical Engineering Parameters								
		Type of Soil	Average CPT Value (kg/cm^2)	Average SPT Value	Void Ratio	Cohesion (kN/m^2)	Unit Weight(kN/m^3)	Angle of Internal Friction	Estimated modu Elasticity(kN/m^2)	Poisson Ratio
0.00 – 1.50	1.50	Loose Silty Sand	14	4	0.80	-	16.50	28^0	7,318	0.3
1.50 – 14.50	13.00	Medium dense Silty Sand	62	14.0	0.65	-	17.50	31^0	15,766	0.3
14.50 – 16.00	1.50	Soft Silty Sandy Clay		4.0	0.81	35.0	18.60	5^0	5,947	0.5
16.00 – 21.00	5.0	Medium dense Silty Sand		21.0	0.55	-	19.50	32^0	24,298	0.3
21.00 – 30.00	9.0	Dense Silty Sand		30.0	0.40	-	20.50	34^0	31,336	0.3

3.5 Geotechnical Engineering Parameters

The summary of geotechnical engineering parameters derived from integration of boring, penetration and laboratory tests results is presented in Table 6. The high void ratios of 0.80 and 0.81 of the loose silty sand and soft silty sandy clay layers respectively are pointers to high settlement which make them technically unsuitable as foundation materials under higher loadings. Also, the low SPT-N value and poor gradation of the upper Silty Sand are indications of susceptibility to liquefaction. Furthermore, table 6.1 shows that the medium to dense silty sand at depths approximately 16.0 – 30.0m have angle of internal friction of 32 – 34^0 pinpointing their higher strength estimates and competency to withstand high foundation loads. Hence, they are the most suitable as foundation materials. Allowing for a maximum permissible settlement of 40.00mm on sand, the loose silty sand layer can only withstand allowable bearing pressure of 29.0kN/m^2 at founding depth of 0.50m which is grossly inadequate for loading from a six storey building. Pile foundation could be installed on the medium dense silty sand layer at depths between approximately 1.50 – 14.0m to generate appreciable foundation pressure; however, care must be taken not to overstress the soft silty clay (14.0 – 16.0m) underneath it. Therefore, considering the loading requirements of a six storey building, the most appropriate foundation should be embedded within the medium dense to dense silty sand due to it relatively high SPT-N values (21 – 34). The ultimate pile capacity and anticipated safe working loads that

can be generated based on assumed diameters and depth is presented in table 7 for both driven and bored piles depending on the loading requirements.

Table 7. Pile Safe Working Load

	Driven Pile				Bored Pile		
Diameter (mm)	Founding Depth (m)	Ultimate Pile Capacity (KN)	Safe Working Load (KN)	Diameter (mm)	Founding Depth (m)	Ultimate Pile Capacity (KN)	Safe Working Load (KN)
300	18.0	1071	428	300	18.0	437	175
400	18.0	1675	670	400	18.0	665	265
500	18.0	2438	975	500	18.0	945	378
600	18.0	3327	1331	600	18.0	1267	507

Conclusions

Geotechnical investigation using boring, Standard and Cone Penetration Tests as well as laboratory engineering methods has revealed that the study area is underlain by loose silty sand (0.0 – 1.50m), medium dense silty sand (1.50 – 14.50m), soft silty sandy clay (14.50 – 16.0m), medium dense silty sand (16.0 – 21.0m) and dense silty sand (21.0 – 30.0m). The good correlation between Standard and Cone Penetration Tests is indicative of the near accuracy of both methods in characterizing the shallow subsurface for geo-engineering applications. The geotechnical laboratory test revealed that the loose silty sand and soft silty clay layers are incompetent as foundation material for a six storey building due to their low shear strength and high void ratios. Due to the high load bearing capacity requirement, deep foundation in form of piles installed within the dense silty sand layer at assumed depth of 18.0m could generate anticipated safe working loads of 428.0kN – 1131.0kN and 175.0kN – 507kN under driven and bored pile respectively with assumed diameters of 300.0 – 600.0mm. If the loading requirements of the structure are above these values, piles of higher diameters and depths should be considered based on the geotechnical parameters presented in this work.

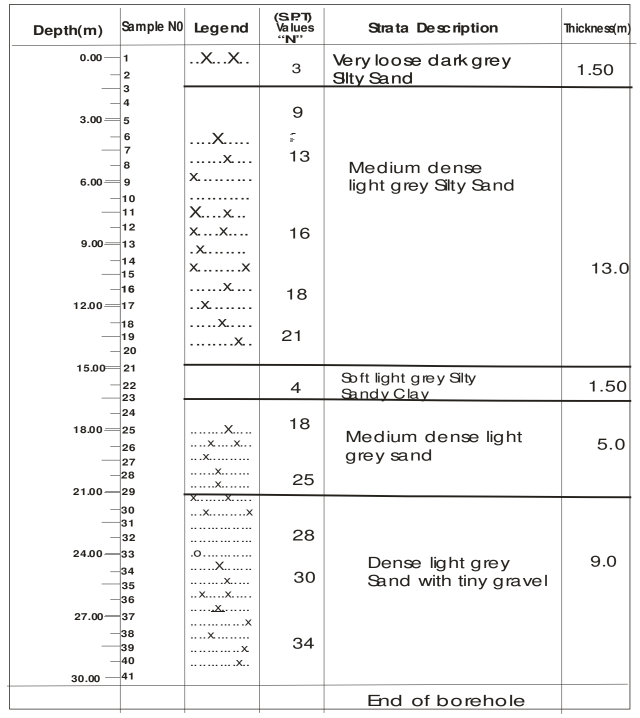

Fig. 7. Log of borehole 1.

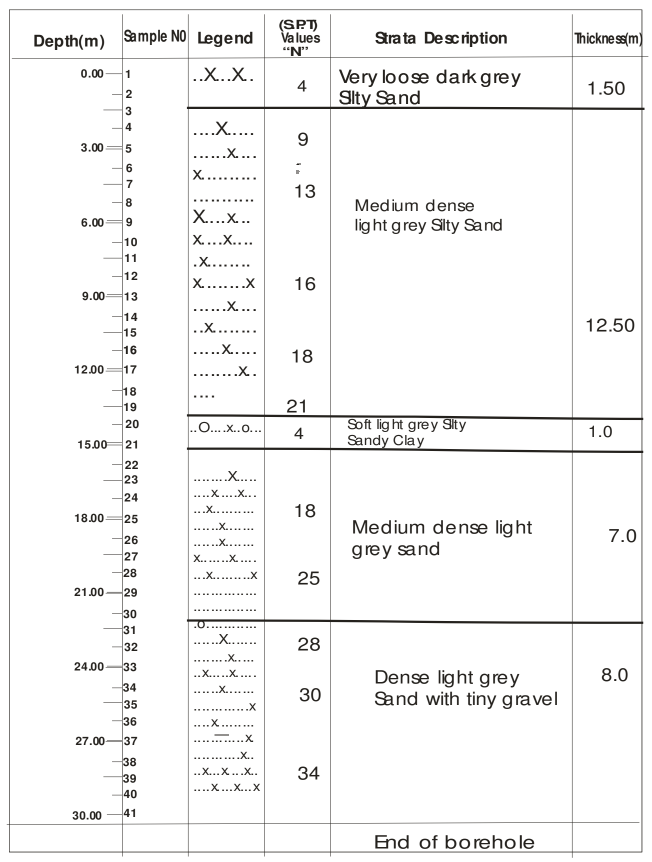

Fig. 8. Log of borehole 2.

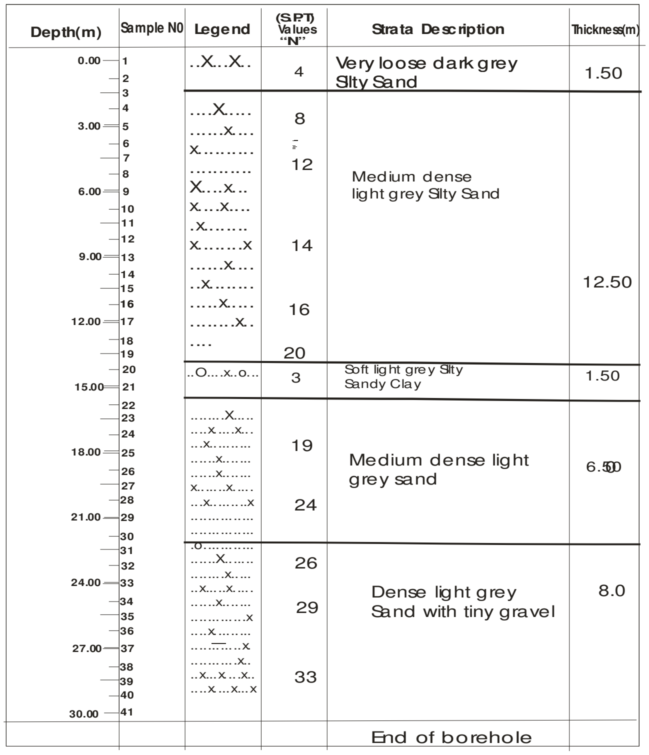

Fig. 9. Log of borehole 3.

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